

CHAPTER 4 ELEMENTS OF DESIGN

4.1 SIGHT DISTANCE

The ability to see ahead is of utmost importance for safe and efficient operation of a highway vehicle. Sight distance can be defined as the length of highway visible to a user of the highway. The four types of sight distances discussed in this chapter are:

- Stopping Sight Distance: the distance required for a vehicle to stop on any type of highway.
- Passing Sight Distance: the distance required to pass a vehicle (applicable only to two-lane highways).
- Decision Sight Distance: the distance needed to make decisions at information sources or hazards.
- Intersection Sight Distance: the distance provided when feasible at intersections to enhance the safety of the facility.

See Chapters 3 and 9 of the AASHTO Green Book for more information on sight distance calculations.

4.1.1 Stopping Sight Distance

Stopping Sight Distance (SSD) is the length of roadway required for a vehicle traveling at the design speed to stop safely before reaching a stationary object in its path. As outlined in Chapter 3 of the AASHTO Green Book, calculations are based on:

- the height of driver's eye of 3.5 ft
- an object height of 2 feet above the pavement
- the passenger vehicle as the design vehicle
- wet pavement conditions

DOTD Design Standards recommend a range of values for SSD. Minimum and desirable values are based on the running speed and the design speed, respectively. As previously noted, Department policy is to use the maximum or desirable value. However, should a situation dictate otherwise, a value less than desirable, but not less than the minimum, could be used with approval. Documentation must be provided.

Adjustments to the calculated lengths should be made if the roadway is on a significant grade, as defined in Chapter 3 of the AASHTO Green Book. AASHTO and DOTD consider the effective length of grade through a crest or sag vertical curve as one fourth of the length of the vertical curve. Therefore, this portion of the vertical curve

would be added to any length beyond the vertical curve when adjusting for significant grades.

4.1.2 Passing Sight Distance

Passing Sight Distance (PSD) is the length of roadway required for a vehicle to safely complete a normal passing maneuver. This value is not included in the Design Standards, and minimum values, as calculated using methods defined in Chapter 3 of the AASHTO Green Book, are appropriate. Lengths are calculated based on the passenger vehicle and an object height of 3.5 ft, equivalent to the height of the standard passenger vehicle.

When setting the horizontal and vertical alignment on a two-lane project, it is essential for the Designer to provide as many areas as possible for safe passing maneuvers. If the horizontal and/or vertical alignments do not allow adequate length for passing, the use of passing lanes may be considered as discussed in Chapter 3 of the AASHTO Green Book.

4.1.3 Decision Sight Distance

Decision Sight Distance (DSD) is the distance required for a driver to:

1. Detect an unexpected or difficult-to-perceive information source or hazard in a roadway environment.
2. Recognize the hazard or its threat potential.
3. Select an appropriate speed and path.
4. Safely initiate and complete the required maneuver.

Adequate stopping sight distance will be provided at all points along the highway. The Designer can further enhance the safety of the facility by providing distances at spot locations sufficient for a driver to negotiate the steps above.

Decision sight distance is usually considerably longer than SSD. The Designer should use engineering judgment to determine where decision sight distance should be provided on a highway, keeping in mind design constraints such as right-of-way and environmental impacts. Chapter 3 of the AASHTO Green Book contains additional discussion of decision sight distance.

4.1.4 Intersection Sight Distance

As shown in Figure 4-1, the Intersection Sight Distance (ISD) is the distance that allows a driver stopped at an intersection to:

- travel across the intersecting roadway by clearing traffic to both the left and the right of the crossing vehicle

- turn left into the crossing roadway, by first clearing traffic to the left, and then enter the traffic stream with vehicles from the right, with minimal effect on the speed of on-coming traffic
- or turn right into the intersecting roadway by entering the traffic stream with vehicles from the left, with minimal effect on the speed of on-coming traffic

Adequate stopping sight distance will be provided at all points along the highway, including intersections. As such, DOTD considers intersections with sufficient SSD to be safe. By providing distances at intersections sufficient to meet the three criteria above, the Designer can further enhance the safety of the facility.

As with DSD, intersection sight distance is usually considerably longer than SSD. Typically, it is not practical to provide intersection sight distance due to other constraints, such as right-of-way impacts, environmental concerns, etc. Intersection sight distance is based on a height of eye of 3.5 ft and a 3.5 ft height of object. Chapter 6 of this manual contains additional discussion of intersection design, and Chapter 9 of the AASHTO Green Book contains additional discussion of intersection sight distance and procedures for calculating distances.

4.2 HORIZONTAL ALIGNMENT

The location and alignment selected for a highway are influenced by factors such as physical controls, environmental considerations, economics, safety, highway classification, and design policies. The horizontal alignment cannot be finalized until it is coordinated with the vertical alignment and cross sectional elements of the roadway.

DOTD typically uses horizontal curves for all deflections in an alignment. However, in special situations, such as roadway reconstruction on an existing alignment, angles with deflections less than or equal to 17-minutes ($0^{\circ}17'$) are allowed with no horizontal curvature. In Chapter 3 of the AASHTO Green Book, the section entitled "General Controls for Horizontal Alignment" lists additional considerations in setting horizontal alignments.

4.2.1 Types of Curves

1. Circular Curves: DOTD typically uses the arc definition of the circular curve. Under this definition, the curve is defined by the radius or by the degree of curve (D°), which is the central angle formed when two radial lines at the center of the curve intersect two points on the curve that are 100 ft apart, measured along the arc of the curve. Additional circular curve information and formulas are shown in see Figure 4-2.
2. Compound Curves: For open highways, compound curves between connecting tangents should be used only where existing topographic controls make a single simple curve impractical. When compound curves are used, the radius of the flatter curve should not exceed the radius of the sharper curve by more than fifty percent (a ratio of 1.5:1) as shown in

Figure 4-3. A ratio of 1.75:1, as per AASHTO (2.0:1 absolute max) may be used on interchange ramps, where compound curves are more common.

This criterion only applies when going from a flatter curve to a sharper one. Therefore, on one-way roadways, the ratio may be exceeded when going from a sharper curve to a flatter one in the direction of travel.

3. Reverse Curves: Any abrupt reversal in alignment should be avoided. A reversal in alignment can be suitably designed by including a sufficient length of tangent between the two curves to provide adequate superelevation transition lengths (see Figure 4-4). See Section 4.6.3 for additional discussion of superelevation transition lengths. For roadways with design speeds less than 45 mph, a minimum tangent of 100 ft should be provided between reverse curves, even if superelevation is not required. On higher-speed roadways, curves that do not require superelevation are so flat that no tangent between the curves is necessary. Where roadways are located alongside bayous and levees, the meandering character and physical constraints of these features make it impractical to completely eliminate all reverse curves separated by short tangents. Design exceptions may be warranted in such cases.
4. Spiral Curves: Spiral curves are not used, except in special cases. For overlay or widening projects, spirals are permitted to remain. For roadways to be re-constructed, spiral curves are replaced with simple curves, unless existing property improvements or other controls make this impractical.
5. Broken-back Curves: Successive curves in the same direction that are separated by a short tangent are known as broken-back curves (see Figure 4-5). DOTD defines this short tangent as one with a length less than $15v$, where v is the design speed in mph. Broken-back curves are very undesirable from an operational and appearance standpoint. Every effort should be made to avoid this type of alignment if possible by separating curves in the same direction with a minimum tangent length of $15v$, preferably by the desirable length of 1500'. As noted in No. 3 above, it may not be feasible or practical in some situations to completely eliminate broken-back curves.
6. Curves with Small Deflection Angles: A short horizontal curve with a small deflection angle (less than 5°) may appear as a kink in the roadway. Where appropriate, this undesirable appearance can be avoided by using curve lengths in excess of that computed by:

$$L = 1000 - (100) (\Delta)$$

where: L = curve length in feet

Δ = deflection angle in degrees

7. Minimum Length of Horizontal Curve: The minimum length of horizontal curve required on rural roadways should be $L_{\min}=15v$ (where v is the

design speed in mph). For new construction and reconstruction projects on collectors and local roads and streets, the Designer should attempt to provide minimum lengths of at least 500 ft. However, longer curves than those specified above may be necessary in order to provide a minimum of one-third of the curve in full superelevation.

4.2.2 Pavement Widening on Curves

On modern highways and streets with 12-ft lanes and high-type alignments, the need for widening on curves has decreased considerably in spite of high speeds. In most cases, degrees of curvature and pavement widths established in the Design Standards preclude the necessity of pavement widening on roadway curves. However, for some conditions of speed, curvature, and width, it remains necessary to widen pavements. Widening should be evaluated at locations including:

- low-speed roadways with near maximum curvature
- ramps
- connecting roadways
- where curves sharper than those specified in the Design Standards are used

For additional discussion and widening values, see Chapter 3, "Pavement Widening on Curves," in the AASHTO Green Book. Widening less than 2 ft is considered impractical and will not be required.

4.2.3 Lane Width Transitions

As discussed in Section 4.2.2, lane width transitions can occur at locations where widening must be developed in curves and at connections to existing pavement, such as occurs at the back of a turnout of an intersecting road. Required taper lengths for open highways can be calculated as:

- $L = (w)(s)$, for design speeds of 45 mph or higher
- $L = \frac{(w)(s)^2}{60}$, for design speeds less than 45 mph

where: L = distance needed to develop widening (ft)
 w = width of widening (ft)
 s = design speed (mph)

Travel lane tapers at turnout connections are typically accomplished at a rate of 20 ft for every 1 ft of widening width required. Shoulder widths at turnouts are typically transitioned within the length required for the roadway taper.

4.2.4 Transition in Number of Lanes

Lane transitions commonly occur where:

- a two-lane roadway merges into a four-lane divided roadway

- a two-lane roadway merges into a five-lane urban roadway
- or a four-lane divided roadway merges into a five-lane urban roadway

As shown in figures 4-6 through 4-8, degrees of curvature appropriate for the design speed of the roadway should be used for these transitions, except in the case of lane drops. The degree of curve should be limited so that no more than reverse crown (2.5 percent) superelevation is required on the lanes. Superelevation and superelevation transitions are discussed in Section 4.6.

A lane drop occurs when transitioning from a four-lane divided roadway to a two-lane roadway in the direction of travel. In these situations, a straight-line taper is preferred, using the appropriate formula in Section 4.2.3 to determine the taper length required.

A transition in the number of lanes also occurs at intersections when developing turn lanes. Design procedures for turn lane development at intersections are discussed in Section 6.2.2.

4.2.5 Miscellaneous

Alignments with excessive curvature and/or a poor combination of curves limit capacity and detract from a pleasing appearance. In addition, they cause economic losses because of increased travel time and operational costs. An alignment should be as direct as possible and remain consistent with the topography, while preserving developed properties and community values. A flowing alignment that generally conforms to the natural contours is preferable to an alignment that slashes through the terrain with long tangents.

In general, the number of short curves should be kept to a minimum. A winding alignment composed of short curves should be avoided because it usually creates erratic operation. Although the aesthetic qualities of a curving alignment are important, long tangents are required on two-lane highways in order to provide passing sight distance along as much of the highway as feasible. A consistent alignment should always be sought. Sharp curves should not be introduced at ends of long tangents. Likewise, sudden changes from areas of flat curvature to areas of sharp curvature should be avoided. Where sharp curvature must be introduced, it should be approached by successively sharper curves from the generally flat curvature, if possible.

Chapter 3 of the AASHTO Green Book discusses additional controls for horizontal alignments.

4.3 VERTICAL ALIGNMENT

DOTD uses design controls for crest and sag vertical curves based on stopping sight distance (SSD). In Chapter 3 of the AASHTO Green Book, minimum and desirable values based on running speed and design speed, respectively, are shown for rate of vertical curvature (K). DOTD has adopted the desirable or maximum values as the

norm. If environmental or economic factors make the use of maximum values undesirable, lesser values (but not less than the minimum) may be used. See Section 2.2 for discussion of design standards.

DOTD typically uses symmetrical parabolic vertical curves at changes in grade (see Figures 4-9 and 4-10). An exception to this is at spot locations, such as overlay transitions, where breaks of up to 0.4 percent and 0.6 percent are allowed, for design speeds of 60 mph and 50 mph, respectively, without the placement of a vertical curve. Maximum breaks allowed at spot locations with lower design speeds should not exceed 1.2 percent, where necessary. In such locations, the vertical transition rate is typically 1 ft in 100 ft (see Figure 4-11). Eccentric (unsymmetrical) vertical curves, as shown in Figure 4-12, are rarely used, but occasionally are required to fit specific site conditions.

Chapter 3 in the AASHTO Green Book lists additional considerations in setting vertical alignments.

4.3.1 Minimum Grades and Vertical Curve Lengths

1. **Uncurbed Pavements:** For uncurbed pavements, longitudinal grades may be flat (zero percent). As with horizontal curves, when changes in grade are required, long, flat vertical curves with lengths in excess of the minimum are generally preferred. On rural roadways, the minimum length of vertical curve will be the longer of either 300 ft or that required by the formula $LVC = KA$, where K is the rate of vertical curvature and A is the algebraic difference in grades (in percent). On spot replacement projects where these lengths are not feasible, the minimum length of vertical curve is $LVC = 3v$, where v is the design speed in mph. Example 4-1 shows a sample calculation of LVC for rural conditions.
2. **Curbed Pavements:** For curbed pavements with subsurface drainage, a minimum longitudinal grade of 0.4 percent, positive or negative, is used. The minimum length of vertical curve should be set based on K values in the AASHTO Green Book and the $LVC = 3V$ criteria. However, the K value provided should not exceed 167 in order to maintain adequate drainage of the pavement. For minimum grades of 0.4 percent, this equates to a maximum vertical curve length of 133.6 ft. This value is normally rounded to a 135 ft vertical curve, since the 1.4 ft difference is insignificant. Examples 4-2 and 4-3 show sample calculations of vertical curve lengths with curbed pavements.

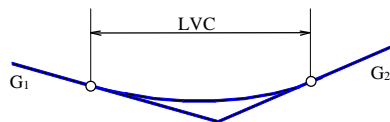
Also, in areas of superelevation transition, gutter grades may be critical and should be checked to ensure proper drainage of the pavement.

EXAMPLE 4-1: Minimum Length of Sag Vertical Curve (Uncurbed)

Assume the following:

- Entrance Grade (G_1) = -1.0%

- Exit Grade (G_2) = +3.0%
- Design Speed = 60 mph
- Rural, Uncurbed Pavement

**Solution**

From Table 3-36 in the AASHTO Green Book, the desirable K value for a 60 mph design speed is 136. $LVC = KA$, where A is the algebraic difference in grades ($G_2 - G_1$) in percent.

$$LVC = (136)(3 - (-1)) = (136)(4) = 544 \text{ ft}$$

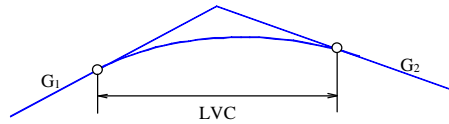
This is greater than the 300 ft minimum requirement, therefore use **544 ft.**

Note: This is the desirable length of vertical curve required for these conditions. The Designer is encouraged to use a longer vertical curve if conditions permit.

EXAMPLE 4-2: Minimum Length of Crest Vertical Curve (Curbed)

Assume the following:

- Entrance Grade (G_1) = +0.4%
- Exit Grade (G_2) = -1.0%
- Design Speed = 45 mph
- Urban, Curbed Pavement

**Solution**

From Table 3-34 in the AASHTO Green Book, the desirable K value for a 45 mph design speed is 61. $LVC = KA$, where A is the algebraic difference in grades ($G_2 - G_1$) in percent.

$$LVC = (61)(-1 - (+0.4)) = (61)(-1.4) = -85.4 \text{ ft (ignore sign)}$$

Checking to ensure that the 3V criteria is met:

$$LVC (\text{min}) = (3)(45) = 135 \text{ ft}$$

85.4 is less than 135, therefore **use 135 ft**

Note: This is the desirable length of vertical curve required for these conditions. This length is typically rounded to an even 10 ft increment. The Designer may use a longer vertical curve if conditions permit. However, to maintain adequate drainage, the curve should not have a K-value in excess of 167.

4.3.2 Spacing Between Vertical Curves

A tangent distance is not required between the end of one vertical curve and the beginning of the next. In addition, no criterion has been developed for setting a minimum distance between Points of Vertical Intersection (PVIs) for vertical curves. However, the number of PVIs in the grade should be limited to the minimum that would provide consistency with the terrain and reasonable earthwork economy. PVIs to introduce small grade changes in rolling and hilly terrain should be avoided. The elimination of these small grade changes not only improves the roadway appearance, but, in many cases, also provides passing sight distance, which may not have been available otherwise.

4.3.3 Earthwork Balance

In establishing the vertical alignment, the Designer should strive for a reasonable balance between general excavation and embankment. Good engineering practice should be used in balancing cuts and fills, considering such factors as maintenance of traffic, environmental impacts, and right-of-way requirements. Also, developing areas with suitable passing sight distance or maintaining the roadway above the high water elevation, as discussed in the following section, may control the Designer's ability to obtain a reasonable earthwork balance.

4.3.4 Minimum Grade Elevation Above High Water

One major factor in a roadway's durability is minimizing the amount of moisture in the base and pavement. Generally, the lowest point of the roadway surface should be determined by adding 1.5 ft to 2.0 ft (1.0 ft of freeboard plus 0.5 ft to 1.0 ft to allow for losses created by drainage structures) to the design high water elevation. See the DOTD [Hydraulics Manual](#) for additional information pertaining to setting the roadway grade.

4.3.5 Miscellaneous

Other factors that influence roadway grades include:

- control points at the beginning and end of the project
- vertical clearances for drainage structures
- intersecting railroads
- intersecting roads and streets
- existing bridges to remain
- proposed new bridges, etc.

If it is impractical to match the elevation of an intersecting road, the crossroad should be reconstructed for a suitable distance using adequate vertical curve lengths to make the grade adjustment. Typically, a maximum difference of 5 percent is allowed

between the roadway cross-slope and the approach intersection grade when the intersection is at a stop condition. For signalized intersections, the maximum break allowed is 2.5 percent.

For vertical clearance criteria, such as at highway and railroad overpass structures, see Section 5.4. See Section 6.8 for additional discussion of grades at railroad crossings.

4.4 COMBINED HORIZONTAL AND VERTICAL ALIGNMENTS

Horizontal and vertical alignments are permanent design elements that warrant thorough study. Horizontal and vertical alignments should not be designed independently, but should complement each other. Poorly designed combinations can negate the benefits and aggravate the deficiencies of each. However, a well-designed combination, in which horizontal and vertical alignments work in concert, increases highway usefulness and safety, encourages uniform speed, and improves appearance. Usually, this can be achieved without additional cost.

4.4.1 Aesthetics

Coordination of the horizontal and vertical alignment can result in a highway that is visually pleasing. This can be achieved in several ways. A sharp horizontal curve should not be introduced at or near the low point of a sag vertical curve, which produces a distorted appearance. On divided roadways, a variable width median, and separate horizontal and vertical alignments can be used. Also, if possible, every effort should be made to line up Points of Vertical Intersection (PVIs) with horizontal Points of Intersection (PIs) and to maintain consistency between the horizontal and vertical curve lengths (see Figure 4-13).

4.4.2 Safety

Sharp horizontal curves should not be introduced at or near the top of a pronounced vertical curve, since the driver cannot perceive the horizontal change in alignment. Also, sharp horizontal curves should not be introduced at or near the low point of a sag vertical curve, since vehicles, particularly trucks, are traveling faster at the bottom of grades.

4.5 SPECIAL DESIGN ELEMENTS

4.5.1 Drainage

Design criteria and procedures for rural and urban drainage are contained in the Department's Hydraulics Manual. In general, the drainage design on a project should not change existing drainage patterns. Some additional aspects to consider are:

1. Rural:

- a. Where the roadway is to be built on a fill section, the use of parallel ditches is important to control flow of water along the toe of the slope, particularly along the upstream side of the embankment (see Figure 4-14).
- b. In flat terrain, ditch grades 0.1 percent or steeper are commonly used. Ditch grades less than 0.1 percent should not be used.
- c. As a general guide, ditch grades in flat terrain should be set to limit the ditch depth to less than 4 ft to 5 ft below natural ground.
- d. Where ditches are required, the minimum ditch flow line should be 2 ft below the lowest part of the base course.
- e. Lateral drainage channels should be reviewed to determine if any channel relocations will be made or if skewed pipe crossings will be required. This could consist of a review of the surveyed outfalls and/or a field review of the outfalls.
- f. Pipes crossing beneath state highways and paved intersecting roads should be designed as cross drain structures. Similarly, if drainage from a cross drain flows along a parallel roadside ditch to a lateral outfall channel, pipes beneath driveways crossing the ditch should be designed like the upstream cross drains.
- g. Mitered pipe headwalls called safety end treatments should be provided at locations where existing cross drain and/or side drain structures must remain within the clear zone. This commonly occurs in rural locations where two lanes are added to the existing two lanes. Safety end treatments should also be provided at locations where it is impractical or impossible to place the ends of new drains beyond the clear zone.

However, safety end treatments should be evaluated for existing major cross drain structures (culverts larger than 54 inches or box culverts larger than 4 ft x 4 ft), since the safety end treatment could reduce the hydraulic capacity of the structure and could cause flooding near the structure. The designer should consider extending major cross drain structures beyond the required clear zone.

2. Urban:

- a. Careful review of overland flow should be made to determine the impact on the roadway. When overland flow is excessive, the flow should be trapped behind the curb by raising the grade and/or by constructing a swale ditch behind the curb. There, the flow can be collected in drop-inlets tied to the storm drain system.
- b. An analysis and review should be made where horizontal curves and sag vertical curves are in combination. In some situations, particularly in superelevation transitions, this results in areas of trapped water on the roadway surface, commonly called “bird baths.” These areas can create safety hazards and be detrimental to the integrity of the pavement. In addition, they are visually unpleasant to

the driver. Adjustments to the curves will be necessary to eliminate this situation.

- c. Outfalls in urban areas should also be evaluated as noted in No. 1.e above. DOTD policy in urban areas is to only improve outfalls to the right-of-way line. The local governing authority will be responsible for the construction and maintenance of any improvements beyond the right-of-way, and they should be so notified.
- d. When developing the storm drain drainage layout, drainage systems along both sides of the roadway are preferred over a single system running along one side with pipes crossing the roadway intermittently.
- e. Catch basins should be located at all low points of sag vertical curves and upstream of drives and turnouts.
- f. The presence or absence of a sanitary sewer collection system is a critical issue when a new storm drain system is proposed. Department policy states that a sanitary sewer connection cannot be made to a DOTD storm drain system (see EDSM 1.1.1.6). If a sanitary sewer collection system does not exist, one of the following may be considered:
 - i. Use an open ditch typical section.
 - ii. Advise the local governing authority of the need for a sanitary sewer collection system. Also, advise that the project will be delayed or cancelled if a system is not provided.

4.5.2 Erosion Control

Each project will contain both temporary and permanent erosion control measures, as further described below.

1. Temporary erosion control items consist of:

- hay bales
- settling basins
- temporary seeding
- check dams
- embankment drains
- silt fencing
- embankment berms

Pay items will usually be included for one or more of the above temporary items. The Hydraulics Manual and Standard Plan EC-01 provide details and guidelines on applications of erosion protection measures. Additional guidelines for the placement of temporary erosion control measures are described in the following sub-sections.

- a. Silt Fencing: Silt fencing is a temporary sediment barrier consisting of a filter fabric supported by posts as it stretches across an area. Its

purpose is to intercept and detain small amounts of sediment. A few basic design guidelines for the location of silt fencing are:

- the outer boundaries of all fill sections
- where erosion would occur due to sheet flow or from small, shallow swale ditches
- where the maximum drainage area behind the silt fence is one fourth of an acre per 100 ft of silt fence length
- where the maximum slope length behind the barrier is 100 ft
- where the maximum gradient behind the barrier is 2:1
- not in live streams or in ditches or swales where flows exceed one cubic foot per second (1cfs)

b. Straw and Hay Bale Barriers: A straw or hay bale barrier is a temporary sediment barrier consisting of a row of entrenched and anchored bales of straw or hay. A few basic design guidelines for locating a straw or hay bale barrier are:

- where erosion would occur in the form of sheet and rill erosion
- minor swales and ditches where the maximum drainage area is 2 acres
- where the effectiveness is required for less than 3 months
- not in live streams or in ditches or swales where there is a possibility of a washout

c. Mulches: Mulches are mats of material placed on the soil surface to prevent erosion. They protect the soil surface from raindrop impact and reduce the velocity of overland flow. Mulches can be organic or synthetic. A few basic guidelines for the use of mulches are:

- on soil areas when protection of the soil surface is desired and temporary or permanent stabilization is not feasible
- in conjunction with temporary seeding

d. Check Dams: A check dam is a small temporary dam constructed across a swale or drainage ditch. The purpose of this measure is to reduce the velocity of concentrated storm water flows, thereby reducing erosion of the swale or ditch. The check dam will trap small amounts of sediment generated in the ditch itself. However, it should not be used as a sediment-trapping device. A few basic design guidelines for the use of a check dam are:

- small open channels which drain 10 acres or less
- not in live streams
- temporary ditches or swales that, because of their short length of service, cannot receive non-erodible lining
- permanent ditches or swales that will not receive a permanent lining for an extended period of time

- temporary or permanent ditches or swales that need protection during the establishment of grass linings
 - a minimum of one check dam in the side ditch at each cross drain site
 - continuous grades, if required, check dams should be placed in side ditches at a maximum of 200 ft apart
- e. Temporary Stone Entrance and/or Wash Rack: A temporary stone entrance is a stone-stabilized pad located at every point where construction traffic leaves the construction site and enters a public road. Its purpose is to reduce the amount of mud transported onto public roads. If the action of the vehicle traveling over the gravel pad is not sufficient to remove the majority of the mud, then the tires must be washed before the vehicles enter the public road. A few basic design guidelines for the use of stone construction entrances and wash racks are:
- the stone layer must be at least 6 inches thick
 - the stone shall conform to Section 1003.04 of the Standard Specifications
 - the length of the pad must be at least 75 ft and it must extend the full width of the vehicular ingress/egress
 - a geotextile fabric beneath the stone is required in accordance with specifications for fabric used beneath rip-rap
 - when a wash rack is necessary, provisions must be made to intercept the wash water and trap the sediment before it is carried off the site

2. Permanent erosion control items consist of:

- seeding
- vegetative mulch
- flexible or rigid revetment
- energy dissipaters
- and erosion control matting

The Hydraulics Manual provides specific levels of permanent channel erosion protection.

3. Under the Environmental Protection Agency's (EPA's) regulation of storm water discharges, the National Pollution Discharge Elimination System (NPDES) General Permit requires that the discharges from all construction sites be managed to prevent pollutants from entering waters of the United States. To accomplish this goal, DOTD prepares the Storm Water Pollution Prevention Plan (SWPPP) at least 45 days prior to the beginning of construction and submits it to the Louisiana Department of Environmental Quality (DEQ). Typically, this plan is used to prepare the erosion control table, which delineates the type and location of the

required erosion control measures (see Section 8.2.5 and Figure 8-25). Copies of the NPDES form are available in the Road Design Section.

4.5.3 Landscaping (Not Included)

4.5.4 Rest Areas (Not Included)

4.5.5 Lighting (Not Included)

4.5.6 Utilities

Utilities located within the limits of construction for the roadway and drainage structures of a project will usually require relocation, adjustment, or encasement. The original field survey indicates the location, size, elevation, type, and owner of each utility identified during the survey. The District Utility Representative uses the plans and cross-sections to inform the utility owners of the impacts to their facilities.

The Designer should be aware that the relocated utilities will normally be accommodated within the required right-of-way. This should be considered in setting required right-of-way limits (see Section 5.11.5). Sewer force mains are treated as a utility and are not allowed to remain under the roadway pavement. The utility owner can move the line, or, if the owner furnishes the relocation design and specifications, the Department will include the required items in the contract. The utility owner may be required to reimburse the Department for the relocation costs.

Generally, gravity sewer mains are the only utility allowed to remain in place. Gravity sewer lines must be evaluated on a case-by-case basis to determine if they can remain in place. Any adjustments or modifications required will be addressed in the plans. Any conflicts with the new storm drain system will be resolved as directed in the Department's Hydraulics Manual. If a local municipality wants to replace an existing gravity sewer line or to install a new one as part of the roadway construction project, it will be required to furnish the appropriate design, details and specifications. The municipality will also be required to reimburse the Department for the cost of the sewer materials and installation.

4.5.7 Traffic Control Devices

Both temporary and permanent traffic control devices are usually included in the plans of construction projects. Typical traffic control devices consist of items such as signs and barricades, signals, and pavement markers, as described below:

1. Signs and Barricades: Details and placement of temporary construction signs and barricades are shown in DOTD's Traffic Control Details. Temporary construction signs and barricades are either shown on the Minimum Construction Signing Sheets or combined with the sequencing on Minimum Construction Signs and Sequence of Construction Sheets

(see Section 8.2.18). Permanent traffic control signs (speed limit signs, curve signs, stop signs, etc.) are not usually included in the contract, but are installed by district forces prior to the project being accepted and/or opened to traffic.

2. Signals: Traffic signals are included in the plans if the project design requires the existing signals to be modified (see Section 8.2.20). New signal locations will not be added unless the warrants for installing signals are met. The Traffic Engineering Section will conduct a warrant analysis upon request to determine if an intersection can be signalized (see EDSM VI.3.1.1 and Section 6.3).
3. Pavement Markings and Markers: Both temporary and permanent pavement markings are included on most projects. Temporary pavement markings will be used to provide guidance to traffic during various stages of construction (see EDSM VI.5.1.1 and Section 8.2.17). Permanent pavement markings and markers will normally be included in all projects as a contract pay item. A layout plan will be detailed for major projects (see EDSM VI.4.1.1 and VI.4.2.2 and Section 8.2.16). Standard Plan PM-01 shows details concerning the placement of permanent pavement markings and markers on highways.

4.5.8 Maintenance of Traffic

During the construction of most projects, through and/or local traffic is maintained within the project limits. A traffic control plan, along with a suggested sequence of construction, is developed in order to provide for the orderly and safe movement of this traffic and to provide a safe working environment for construction personnel

At the Plan-in-Hand Inspection, District personnel, city officials, and other interested parties review the suggested sequence of construction and provide advice concerning the feasibility of the plan (see Section 8.2.17). After incorporating the comments from the Plan-in-Hand Inspection, Construction Signing Sheets are developed and submitted to the Traffic Engineering Section for review.

1. Flagging Operations: The contractor can sometimes maintain traffic at a particular work area using flagging operations. If additional traffic control is required at a specific site, such as at a bridge, large culvert, or other feature, a detour around the site is necessary. The Designer uses traffic volumes to determine the type of detour required (see Section 4.5.9).
2. Special Sequencing: In some projects, traffic is maintained through the construction area using a specific sequence of construction. If the project typical section allows, a portion of the finished roadway section is constructed adjacent to the existing roadway, traffic is detoured to the new pavement, and the remainder of the roadway section is completed. In some cases, temporary widening will be required to maintain adequate width for traffic and provide sufficient work area for construction. Details for these operations may be included in the Suggested Sequence of Construction (see Section 8.2.17).

3. Off-site Traffic Detours: On some projects, it may be more economical to close the existing roadway and detour through-traffic over other state routes, due to the expense of on-site diversions or the extent of new construction. In order to use this scheme, another suitable state route(s) must be available. If this option appears feasible, the District will discuss the road closure with local officials and provide a recommendation concerning the road closure. In some cases, a cost estimate of the on-site diversion(s) may be prepared by the Designer and submitted to the Office of Planning and Programming so that a Benefit-Cost Study can be prepared to evaluate all options.

In a few unusual situations, parish roads are sometimes used as detours. However, this requires the approval of the parish, an in-depth evaluation of the road, and possible repairs to the parish road before starting the project, during construction, and after completing the project. The ultimate cost may make this course of action unacceptable.

The decision to use an off-site detour should be made as early as possible in the design process. If this has not been determined prior to the Pre-Design Conference, a decision should be made at that time.

4. On-site Diversions: Another option available to maintain traffic during construction is to use on-site diversions around specific areas of construction. On-site diversions are discussed in detail in Section 4.5.9.

4.5.9 Diversions

1. General: The Department makes a concerted effort to maintain traffic through construction areas with as little delay and inconvenience to the traveling public as possible. To accomplish this, diversions built adjacent to the construction may be necessary. Diversions are costly, require excessive construction servitude, and are undesirable if other alternatives exist. Before calling for detour roadways, Designers must make every effort to consider all alternatives, such as closing the site to through-traffic during construction, split-slab construction of new bridges, offset alignment of the proposed construction, etc. When a bridge is involved, the decision should be made jointly between the Road and Bridge Design Sections and the District.

As mentioned in Section 4.5.8, No. 3, the District usually makes the initial recommendation whether or not the road can be closed to traffic. Based on this recommendation, the Designer places diversions, sequencing and/or road closures in the preliminary plans as required to complete the project. The Designer should establish the diversions taking into consideration such factors as:

- traffic volume and composition
- design speed
- minimum safety standards
- in-service time of the detour

- drainage
- surfacing requirements
- and right-of-way restrictions

These recommendations will be confirmed or modified by the Plan-in-Hand Party. After the Plan-in-Hand Inspection, all necessary details and changes to the diversions and/or sequencing are to be incorporated in the final plans. The project specifications should contain adequate information concerning the contractor's responsibility for construction and maintenance of the diversions.

2. Design Criteria: The Department's suggested design criteria for detours are shown in Figures 4-15, and the geometric details and typical sections are shown in Figures 4-16 through 4-18. It should be understood that these are merely suggested design criteria. The diversion should be designed for a speed appropriate for each site. The Plan-in-Hand Party may make certain modifications to the alignment and/or typical sections of the detours, as they deem necessary, to best fit actual project conditions.

For high traffic diversions, a specific typical section design may be warranted. In such situations, the detour typical section design is requested from the Pavement and Geotechnical Design Unit of the Bridge Design Section. With the request, the Designer should provide traffic data and an estimate of the length of time the detour will be in service.

- a. Types: The current average daily traffic (ADT) volume is used to determine the type of detour road required.

In addition, a three-phase diversion, as shown in Figure 4-19, can be used at certain spot sites to replace or install cross drains. The maximum curvature for these diversions should meet the requirements for the type diversion (Type B, C, or D) applicable to the route. However, the offsets to the centerline can be reduced to the minimum required for proper construction.

- b. Current ADT: The diversion design should be consistent with AASHTO Green Book minimum requirements, to the extent feasible within the scope of the project. Figures 4-16 through 4-18 show the types of diversions suggested for various diversion design speeds. Type B, C, and D diversions should be detailed in the plans and cross sections as outlined in Chapter 8 of this manual.
- c. Minimum Design Speed, Curvature, and Superelevation: Diversion design speed, curvature and superelevation should be selected as appropriate for each specific site.

Type B, C and D diversions, as shown in Figures 4-16 through 4-18 respectively, are designed with curvature that eliminates the need for superelevation based on Method 2 outlined in Chapter 3 of the AASHTO Green Book. The design speed, superelevation and

curvature should be evaluated and modified as needed to fit the specific needs at a site.

3. Drainage Design: Drainage structure openings for Types B, C, and D diversions will be determined by the designer in accordance with the DOTD Hydraulics Manual and will be shown in the plans. The minimum design storm frequency for the diversion is two (2) years. Drainage structures will be sized so that the allowable headwater is at least 6 in. below finished grade of the diversion, unless other controls govern the situation, such as surrounding development or predicted excessive outlet erosion. The diversion should be lower than the main road in order to provide relief in case of a large storm event while the diversion is in place. LADOTD will assume the responsibility for the hydraulic capacity of these diversions, and, in the event the diversions are damaged by flood waters due to insufficient hydraulic capacity, will reimburse the contractor for the costs of restoring the diversions to their original condition.
4. Diversion Length:
 - a. When diversions are used, their lengths will be kept to a minimum, unless the Designer determines that some additional length will make construction easier and more cost effective.
 - b. In general, diversion lengths at bridge sites will not be determined to accommodate the full embankment widening. Instead, construction will be phased in such a way that the embankment can be completed after the bridge has been opened to traffic. However, the diversions must be long enough so that an interim guardrail length of 62.5 ft can be placed before returning traffic to the main roadway.
 - c. If the proposed bridge elevation is considerably higher than the existing structure, the diversions length does not necessarily need to be extended to the project limits or to where the new roadway touches down. In some cases, the diversion alignment can be tied back to the existing roadway at a location where the proposed roadway fill height is less than 3 ft above the existing roadway, if permitted by soil conditions, traffic volume, and other constraints. Upon removal of the diversions road, the remaining portion of the permanent roadway can be completed under traffic.

4.6 SUPERELEVATION

4.6.1 General

When a vehicle travels around a horizontal curve, it is forced radially outward by centrifugal force. When this force becomes too great for a given design speed, the roadway is superelevated to counter it. In Chapter 3 of the AASHTO Green Book, five methods of counteracting centrifugal forces through curves are discussed. DOTD uses

Method 2 for low-speed (less than or equal to 40 mph) urban roadways and detours. Method 5 is used for all other situations.

The AASHTO Green Book gives superelevation requirements for maximum superelevation rates (0.04 to 0.12 ft/ft) for various design speeds (20 to 70 mph). DOTD has designated a maximum superelevation rate of 0.04 ft/ft for urban roadways, 0.10 ft/ft for rural roadways, and 0.08 ft/ft for ramps. Superelevation rates in the AASHTO tables are based on a normal crown of 0.015 ft/ft. Since DOTD uses a normal crown of 0.025 ft/ft (see Section 5.2), adjustments to values shown in the tables are needed at the lower ranges (from NC to RC). In addition, the values for the runoff length in the Green Book tables should not be used. Rather, the transition length should be calculated based on approved DOTD methods (see Section 4.6.3).

It is important to note that in some cases, such as where superelevation transitions fall on bridges, graphical grades can be used to provide smooth edge profiles. In this situation, careful coordination must take place to ensure that the approach roadway and shoulder design match the proposed bridge.

Shoulder rates in superelevated sections are discussed in Section 5.2.2.

4.6.2 Axis of Rotation

The roadway may be rotated about various points on the typical section to achieve superelevation (see Figure 4-20). The below descriptions refer to rotation about points on the pavement. However, because DOTD uses the grading section as its earthwork and cross section control, the actual point of rotation used by DOTD in many cases may be located at a point on the grading section at the bottom of the base course.

1. Inside Edge of Pavement or Gutter Line: For crowned roadways up to five lanes in width, rotation about the inside edge of pavement is generally preferred. In flat terrain, it minimizes problems of setting ditch grades to drain the inside roadway ditch. See Figure 4-21 for an example superelevation diagram.

The outside lane(s) is rotated about the centerline until reverse crown is achieved (0.025 ft/ft). The roadway is then rotated about the inside edge of pavement until full superelevation is achieved.

For curbed sections, rotating about the inside gutter line avoids drainage complications by maintaining a consistent gutter grade.

2. Centerline of Pavement: This method is most commonly used for multi-lane highways with depressed medians greater than 40 ft wide (see Figure 4-22 for an example superelevation diagram). Generally, for this type of facility, separate grades are established for each roadway.

For medians less than 64 ft in width, consideration should be given to setting the centerline grade of the inside roadway lower than the centerline grade of the outside roadway to account for the difference

created with superelevation. This is more pleasing in appearance, and permits more satisfactory pavement slopes for median cross-overs.

3. Median Edges of Pavement: Identical profiles along the median edges of each roadway are preferred for roadways with raised medians greater than four feet in width and depressed medians less than or equal to 40 ft in width. Rotating about the median edge of pavement to achieve superelevation allows the Designer to maintain this relationship. See Figure 4-23 for an example superelevation diagram.

4.6.3 Superelevation Transitions

The following discussions of superelevation transition length and location apply to typical curves. Since requirements at a particular curve on a project may be unique, the Designer may have to use sound engineering judgment to fit the needs of the location while using principles discussed in Chapter 3 of the AASHTO Green Book.

1. Superelevation Transition Length: The superelevation runoff length refers to the length of the highway needed to change the cross-slope at the beginning and end of horizontal curves from a section with adverse crown removed (0 percent pavement slope) to a fully superelevated section, or vice versa. Tangent runoff refers to the length of highway needed to change the cross-slope from a normal crown to a section with adverse crown removed, or vice versa. DOTD combines these values into a total superelevation transition length (see Figure 4-24).

Because the length of superelevation transition will be dependent on the point of rotation, the values for runoff shown in the AASHTO Green Book superelevation tables (Tables 3-17 a and b) should not be used. Rather, DOTD prefers to calculate superelevation transition lengths using methods similar to those shown in Examples 4-4 through 4-9. The calculated total superelevation transition length should be rounded up to reasonably even lengths (typically multiples of 10 ft).

For simplification, DOTD eliminates the intermediate break in the pavement edge profiles shown for the outside edge of pavement at point C in Figure 3-16 of the AASHTO Green Book. By doing this, one constant rate of change in the outside edge profile is created from beginning of the transition to the end. Vertical curve lengths of 100 ft or more are used for all breaks in pavement-edge profiles (see Figures 4-21 through 4-23).

At the Point of Compound Curvature (PCC), the superelevation transition length calculated for the curve requiring the largest superelevation rate will be used. For reverse curves, the total of the calculated values for each curve will be used (the sum of the values).

2. Location of Runoff: For simple curves, 80 percent of the total superelevation transition length is typically placed on the tangent, and the remaining 20 percent is typically placed within the curve. If conditions do

not allow this split, the project coordinator can reduce the percentage of transition outside the curve on a case-by-case basis. For compound curves, the superelevation transition is placed symmetrically about the PCC.

For reverse curves with minimum or near minimum tangent distance between curves, the superelevation transitions may be joined together, without a break in the pavement-edge profiles (see Figures 4-21 through 4-23). In this case, the flat section of pavement cross-slope created during the transition should kept be as short as possible by minimizing the transition length. The Designer should also attempt to keep the flat section from coinciding with a flat area of the longitudinal grade.

3. Transition Length Formula: In keeping with Chapter 3 of the Green Book, DOTD uses the following formula to calculate superelevation transition lengths:

$$L = \frac{(\text{Slope Change})(W) (\text{Lane Factor})}{(\text{Equivalent Maximum Relative Slope})}$$

where: Slope Change is in ft/ft
 W is Lane Width in ft

The maximum relative gradient between the edge of two-lane pavements and the centerline for various design speeds is found in Table 3-15 of the AASHTO Green Book. The most common equivalent maximum relative slopes used by DOTD are:

DESIGN SPEED	MAXIMUM RELATIVE GRADIENT
30 mph	1:152
40 mph	1:172
45 mph	1:185
50 mph	1:200
60 mph	1:222
70 mph	1:250

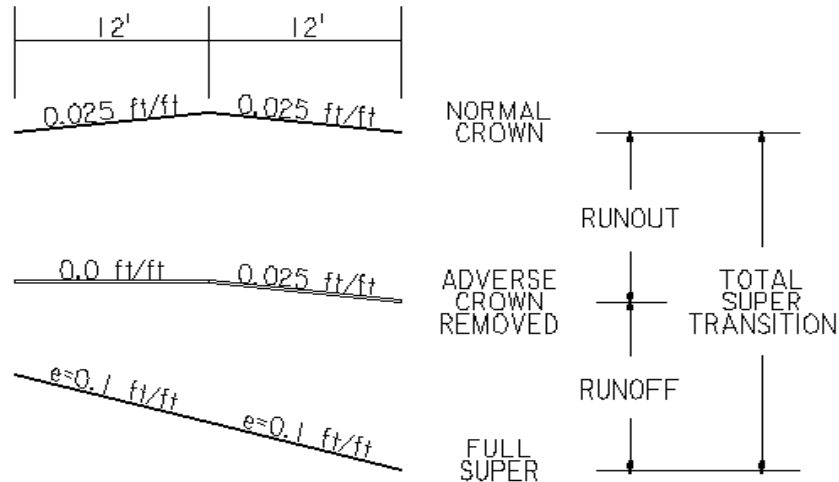
The lane factor is an adjustment for pavements wider than two-lanes and for rotation points other than along the pavement centerline. Lane factors can be found in Table 3-16 of the AASHTO Green Book. The most common lane factors used by DOTD are listed below:

LANE FACTORS		
	ROTATED ABOUT CENTERLINE	ROTATED ABOUT EDGE
2-lane	1.0	1.5
3-lane	1.2	2.0
4-lane	1.5	2.5
5-lane	1.7	3.0
6-lane	2.0	3.5

EXAMPLE 4-3: Superelevation Transition Length 2-Lane Roadway Rotated About Centerline

Assume the following:

- Full super rate (e) = 0.10 ft/ft
- Design Speed = 60 mph
- Lane Width = 12 ft
- Curve to the Right



Total Transition Length

The outside lane is rotated about the centerline from normal crown (+0.025) to full superelevation (-0.1). Therefore, the slope changes from +0.025 to -0.10. Using the tables given in Section 4.6.3, the maximum relative gradient is 1:222 and the lane factor is 1.0.

$$L_{\text{TRANSITION}} = \frac{(-0.10 - (+0.025))(12)(1.0)}{(1/222)} = -333.0 \text{ ft}$$

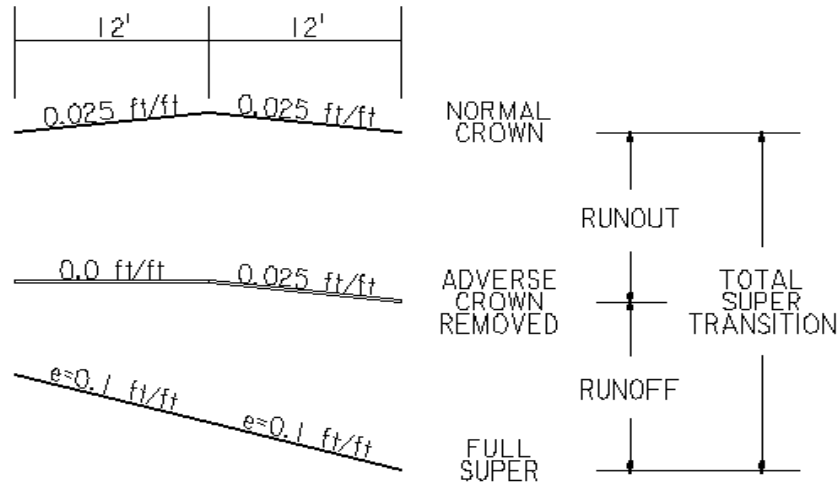
Taking the absolute value and rounding up to the next ten feet, **use 340 ft**.

Using the rule that 80 percent of the transition should be on the tangent and 20 percent inside the curve, 272 ft would be on tangent and 68 ft would be in the curve.

EXAMPLE 4-4: Superelevation Transition Length 2-Lane Roadway Rotated About Inside Edge

Assume the following:

- Full super rate (e) = 0.10 ft/ft
- Design Speed = 60 mph
- Lane Width = 12 ft
- Curve to the Right



Total Transition Length

As in Example 4-4, the outside lane is rotated about the centerline from normal crown (+0.025) to full superelevation (-0.1). Therefore, the slope changes from +0.025 to -0.10. Using the tables given in Section 4.6.3, the maximum relative gradient is 1:222. However, since the rotation is about the pavement edge in this example, the lane factor is 1.5.

$$L_{\text{TRANSITION}} = \frac{(-0.10 - (+0.025))(12)(1.5)}{(1/222)} = -499.5 \text{ ft}$$

Taking the absolute value and rounding up to the next ten feet, **use 500 ft**.

Using the rule that 80 percent of the transition should be on the tangent and 20 percent inside the curve, 400 ft would be on tangent and 100 ft would be in the curve.

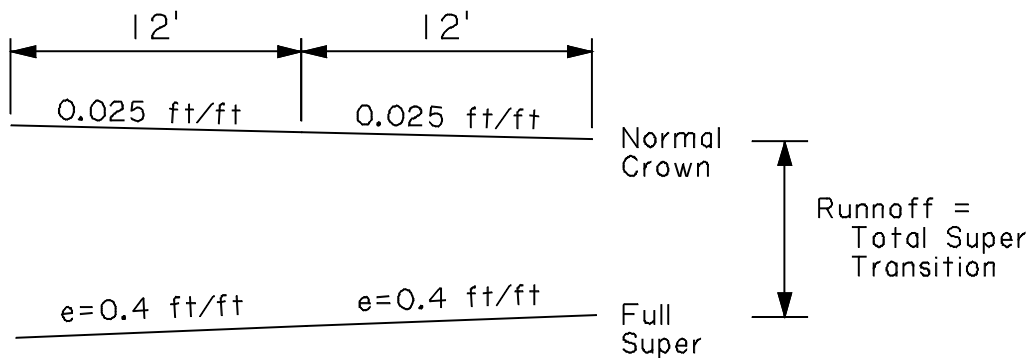
EXAMPLE 4-5: Superelevation Transition Length

4-lane Divided Rural Roadway Rotated About Centerline of Pavement (Median Width > 40 ft)

Assume the following:

- Full super rate (e) = 0.10 ft/ft
- Design Speed = 70 mph
- Lane Width = 12 ft
- Curve to the Left

For a rural divided highway, the superelevation transition is calculated for each roadway independently, and the required lengths are appropriately located as per the discussion in Section 4.6.3. The calculation below is for the right roadway. The requirements for the left roadway would be calculated in a similar manner.



Total Transition Length (Right Roadway)

In this example, both lanes are rotated as a plane from -0.025 to +0.10, but since the rotation is about the centerline, the lane factor is 1.0. The slope changes from -0.025 to +0.1. Using the table given in Section 4.6.3, the maximum relative gradient is 1:250.

$$L_{\text{TRANSITION}} = \frac{(0.1 - (-0.025))(12)(1.0)}{(1/250)} = 375 \text{ ft}$$

Rounding up to the next ten feet, **use 380 ft.**

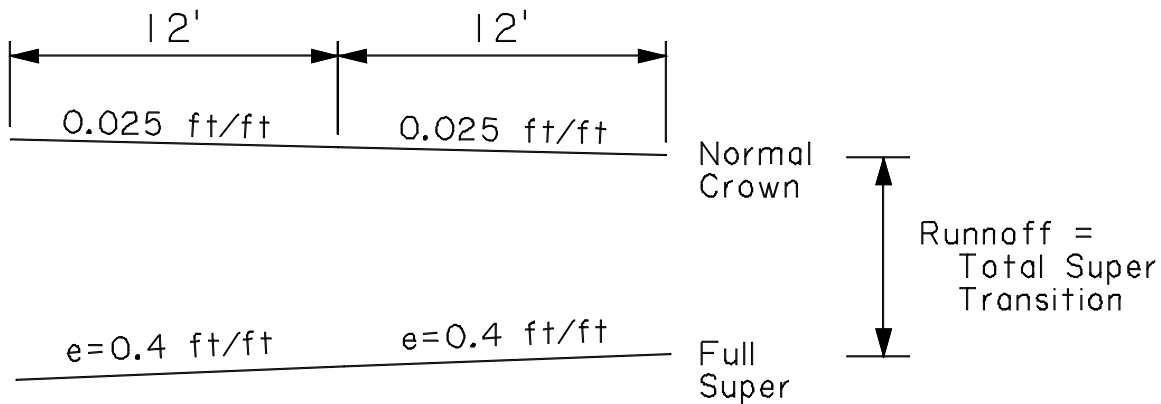
Using the rule that 80 percent of the transition should be on the tangent and 20 percent inside the curve, 304 ft would be on tangent and 76 ft would be in the curve.

EXAMPLE 4-6 Superelevation Transition Length: 4-lane Divided Urban Roadway Rotated About Median Edge of Pavement (Raised Median Width = 20 ft)

Assume the following:

- Full super rate (e) = 0.04 ft/ft
- Design Speed = 45 mph
- Lane Width = 12 ft
- Curve to the Left

As in Example 4-6, the superelevation transition is calculated for each roadway independently, and the required lengths are appropriately located as per the discussion in Section 4.6.3. The calculation below is for the right roadway. The requirements for the left roadway would be calculated in a similar manner.



Total Transition Length (Right Roadway)

In this example, both lanes are rotated as a plane from -0.025 to +0.04. However, the rotation is about the median edge, and therefore the lane factor is 1.5. Using the table given in Section 4.6.3, the maximum relative gradient is 1:188.

$$L_{\text{TRANSITION}} = \frac{(0.04 - (-0.025))(12)(1.5)}{(1/188)} = 220 \text{ ft}$$

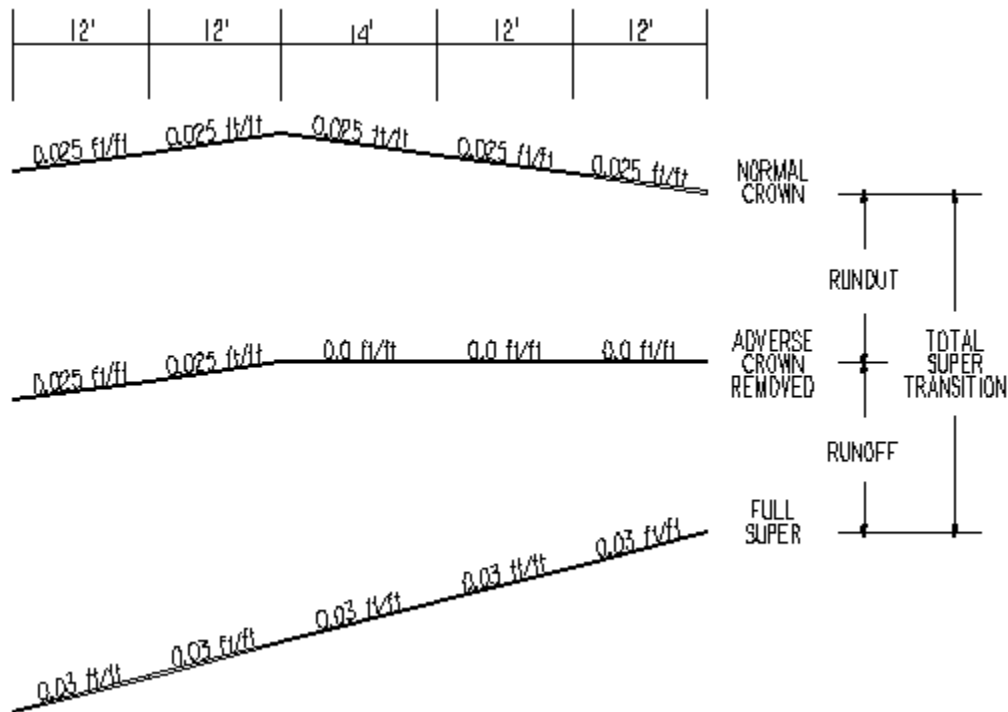
Since there is an even ten feet, no rounding is required; **use 220 ft.**

Using the rule that 80 percent of the transition should be on the tangent and 20 percent inside the curve, 176 ft would be on tangent and 44 ft would be in the curve.

EXAMPLE 4-7: Superelevation Transition Length 5-Lane Roadway Rotated About Pavement Crown

Assume the following:

- Full super rate (e) = 0.03 ft/ft
- Travel Lane Width = 12 ft
- Curve to the Left
- Design Speed = 45 mph
- Continuous Center Turn Lane = 14 ft



Total Transition Length

The AASHTO Green Book recommends that for 6-lane undivided pavements rotated about the centerline, the transition should be twice that required for a two-lane roadway. In this example, the three lanes to the right of the pavement crown are rotated about the crown (the same as the above AASHTO six-lane), and therefore, as shown in the table in Section 4.6.3, the lane factor of 2 will be used. The slope changes from -0.025 to 0.03. Using the tables in Section 4.6.3, the maximum relative gradient is 1:188. The slope of the two lanes left of the crown remains at +0.025 until the right lanes reach +0.025. Then, the left lanes would be rotated with the right lanes to +0.03.

$$L_{\text{TRANSITION}} = \frac{(0.03 - (-0.025))(12)(2.0)}{(1/188)} = 248 \text{ ft}$$

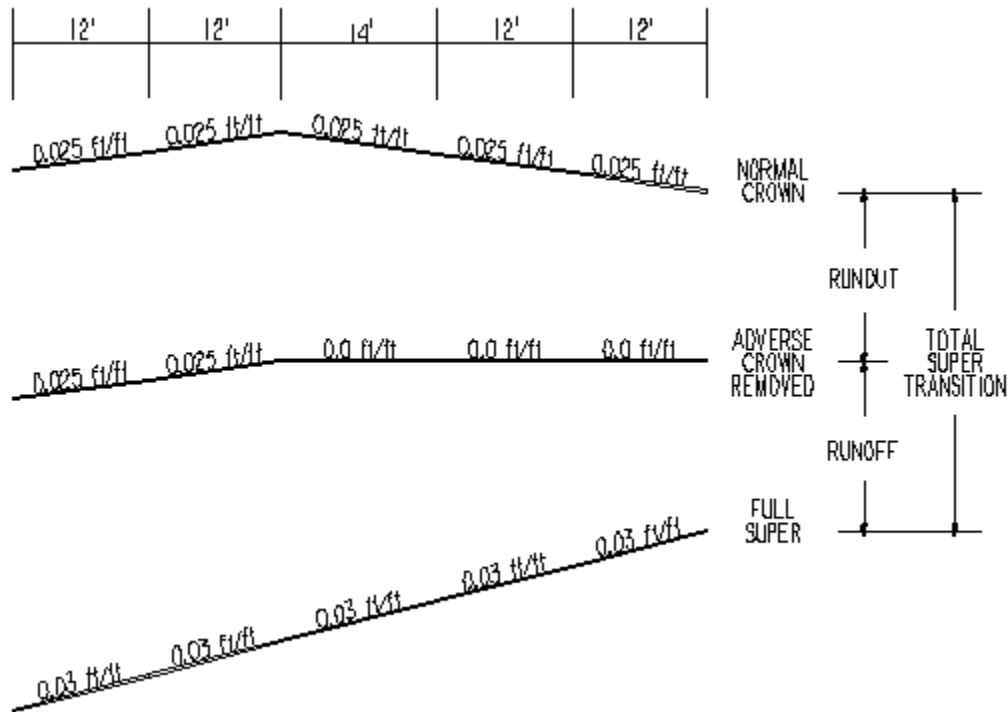
Rounding up to the next ten feet, **use 250 ft.**

Using the rule that 80percent of the transition should be on the tangent and 20percent inside the curve, 200 ft would be on tangent and 50 ft would be in the curve.

EXAMPLE 4-8: Superelevation Transition Length 5-Lane Roadway Rotated About Inside Edge

Assume the following:

- Full super rate (e) = 0.03 ft/ft
- Travel Lane Width = 12 ft
- Curve to the left
- Design Speed = 45mph
- Continuous Center Turn Lane = 14 ft



Total Transition Length

In this example, five lanes will be rotated about the inside edge of pavement. Therefore, as shown in the Table in Section 4.6.3, the lane factor of 3.0 will be used. The slope changes from -0.025 to 0.03. Using the Tables in Section 4.6.3, the maximum relative gradient is 1:188. The slope of the two lanes left of the crown remains at +0.025 until the right lanes reach +0.025. Then, the rotation point shifts to the inside edge, and the entire pavement is rotated as a plane to +0.03.

$$L_{\text{TRANSITION}} = \frac{(0.03 - (-0.025))(12)(3.0)}{(1/188)} = 372 \text{ ft}$$

Rounding up to the next 10ft, **use 380 ft.**

Using the rule that 80percent of the transition should be on the tangent and 20 percent inside the curve, 304 ft would be on tangent and 76 ft would be in the curve.